Chapter 4 Seismic Design and Retrofit — Contents

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4.1 General

The purpose of this chapter is to provide designers with WSDOT seismic design practice and criteria.

Beginning January 2008, WSDOT requires all new bridges, bridge widenings, and retaining walls that have not started design (progressed beyond the Preliminary Plan stage) to be designed in accordance with the requirements of the AASHTO Guide Specifications for LRFD Seismic Bridge Design and as modified by BDM Section 4.2 below. More requirements and design memos may follow with the experience gained from the upcoming designs.

All highway bridges in Washington State are classified as “Normal” except for special major bridges. Special major bridges fitting the classifications of either “Critical” or “Essential” will be so designated by either the Bridge and Structures Engineer or the Bridge Design Engineer.

4.2 WSDOT Modifications to AASHTO Guide specifications for LRFD Seismic Bridge Design

The following items summarize WSDOT's additional requirements and deviations from the AASHTO Guide Specifications for LRFD Seismic Bridge Design:

Article	Subject	WSDOT Requirements
3.1	Applicability of the specifications	AASHTO Guide Specifications is applicable to the design of conventional bridges with span length not exceeding 300 ft. Seismic design requirements of non-conventional bridges, and bridges categorized as essential or critical shall be with the consultation of WSDOT Geotechnical Engineer and Bridge Design Engineer.
3.3	Earthquake Resisting Systems (ERS) Requirements for SDC C & D	WSDOT Global Seismic Design Strategies: Type 1: Ductile substructure with essentially elastic superstructure. This category is permissible. Type 2: Essentially elastic substructure with a ductile superstructure. This category is not permissible. Type 3: Elastic superstructure and substructure with a fusing mechanism between the two. This category is permissible with Bridge Design Engineer's approval.
3.3	Earthquake Resisting Systems (ERS) Requirements for SDC C & D	**Permissible Earthquake Resisting System (ERS), see Figure 3.3-1a:** Types 1 and 3 are permissible. Types 2, 4 & 5 are permissible with Bridge Design Engineer's approval. Type 6 is not Permissible. **Permissible Earthquake Resisting Elements (ERE), see Figure 3.3-1b:** Types 1, 2, 7, 8, 9, 10 &14 are permissible ERE. Types 3, 5, 6, 11, 12 & 13 are permissible ERE with Bridge Design Engineer's approval. Type 4 is not permissible. **Permissible Earthquake Resisting Elements that require Owner's Approval, see Figure 3.3-2:** Types 1 & 2 are permissible ERE with Bridge Design Engineer's approval. Types 3, 4, 5, 6, 7, 8 & 9 are not Permissible. **Earthquake Resisting Elements that are not Recommended for New Bridges, see Figure 3.3-3:** Types 1, 2, 3, & 4 are not Permissible. Permissible ERS and ERE systems with Bridge Design Engineer's approval are applicable to all projects regardless of contracting methods.
3.4	Seismic Ground Shaking Hazard	The procedure used to determine the ground shaking hazard for site class F, critical or essential bridges shall be based on the WSDOT Geotechnical Engineer recommendations.

3.5	Selection of Seismic Design Category (SDC)	All structural designs in Western Washington shall be designed in accordance with SDC C or D. This applies to all structures West of the Cascade Crest (West of MP 157 on SR 20; West of MP 65 on US2; West of MP 52 on I-90; West of MP 69 on SR 410; West of MP 151 on US 12; and West of MP 63 on SR 14). Pushover Analysis shall be used to determine displacement capacity for both SDC C & D. If liquefaction-induced lateral spreading or slope failure that could impact the stability of the bridge could occur, the bridge should be designed in accordance with SDC D, regardless of the magnitude of S_{D1}.
3.6	Temporary and Staged Construction	Design response spectra for temporary bridges and bridges built in staged construction may be reduced by a factor of not more than 2.5. However, it shall be clear in the contract document that structure is designed for reduced response spectra. The provisions of this article apply to temporary bridges and bridges built in staged construction that is considered for not more than 3 years in service.
4.1.2	Balanced Stiffness Requirements	Balanced stiffness requirements and balanced frame geometry requirement shall be satisfied for bridges in both SDC C & D.
4.1.3	Balanced Frame Geometry Requirements	Deviation from balanced stiffness and balanced frame geometry requirements shall be approved by Bridge Design Engineer.
4.2	Selection of Analysis Procedure to Determine Seismic Demand	Analysis Procedures: Procedure 1 (Equivalent Static Analysis) shall not be used. Procedure 2 (Elastic Dynamic Analysis) shall be used for all regular bridges with 2 through 6 spans. Procedure 3 (Nonlinear Time History) may be used where applicable. The time histories of input acceleration used to describe the earthquake loads shall be selected in consultation with WSDOT Geotechnical Engineer and Bridge Design Engineer.
4.9	Member ductility Requirement for SDC C and D	In-ground hinging shall not be considered for drilled shaft foundations. In-ground hinging for pile foundation shall be upon Bridge Design Engineer approval.
4.12.3	Minimum Support Length Requirements Seismic Design Category D	For simple span superstructures, the support lengths shall be 150% of the empirical support length, N, specified by Equation 4.12.2-1
4.13.1	Abutments	Longitudinal restrainers shall be designed in accordance with the requirements of WSDOT BDM Section 4.3.5
5.2	Abutments	Participation of abutment walls in the overall dynamic response of bridge systems during earthquake loading and in providing resistance to seismically induced inertial loads may be considered in the seismic design of bridges when approval is given by the WSDOT Bridge Design Engineer as required in Section 3.3
5.3	Foundation - general	Requirement of foundation modeling method (FMM) shall be based on the WSDOT Geotechnical Engineer's recommendations.

5.6.2	Figure 5.6.2-1	The horizontal axis label of Figure 5.6.2-1 for both (a) Circular Sections and (b) Rectangular sections shall be Axial Load Ratio $\frac{P}{f'_{ce} A_g}$
5.6.3	I_{eff} for Box Girder Superstructure	Gross moment of inertia shall be used for box girder superstructure modeling.
6.3.4	Resistance to Overturning	Revise the resistance factor for overturning of footing to $\phi = 1.0$
6.3.5	Resistance to Sliding	Revise the resistance factor for sliding of footing to $\phi = 1.0$
6.3.6	Flexure	Revise Eq. (6.3.6-1) as follows: $\phi M_n \geq M_u$
6.3.7	Shear	Revise Eq. (6.3.7-1) as follows: $\phi V_n \geq V_u$
6.4.2	Moment Capacity of Pile Foundations	In Eq. 6.4.2-2 change: $M_{p(x)}^{col}$ to $M_{(x)}^{col}$ And $M_{p(x)}^{col}$ to $M_{(x)}^{col}$
6.4.5	Footing Joint Shear SDC C and D	Revise Eq. (6.4.5-11) as follows: $(B_c + D_{ftg})(D_{cj} + D_{ftg})$
6.7.1	Longitudinal Direction requirement	Case 2: Earthquake Resisting System (ERS) with abutment contribution may be used provided that the mobilized longitudinal passive pressure is less than the 0.50 of the value obtained using procedure given in Article 5.2.
6.8	Liquefaction Design Requirements	Liquefaction design requirements shall be considered for bridges in both SDC C & D. Soil liquefaction assessment shall be based on the Geotechnical Engineer's recommendation for each bridge site. • For all bridge foundations with liquefaction identified, structures shall be designed and analyzed for both a non-liquefied and liquefied soil column per the recommendations discussed in Section 6.8 of the Guide Specification for LRFD Seismic Bridge Design. For the liquefied soil analysis case a reduced site-specific response spectrum may be considered along with in-ground foundation element inelastic behavior subject to approval of the WSDOT Bridge Design Engineer. In addition to the above requirements and for site conditions where lateral spreading and downdrag are identified, the design process shall consider these two loading conditions as independent of the seismic inertial lateral loads. Lateral spread forces and gravity loads shall be resisted by foundation elements supported in liquefied soil which shall include reduced axial skin friction resistance. Downdrag and gravity loads shall be resisted by foundation elements considering reduced axial skin friction resistance.
8.4.1	Reinforcing Steel	Only ASTM A 706 reinforcing steel shall be used. Deformed welded wire fabric may be used with Bridge Design Engineer’s approval. Wire rope or strands for spirals, and high strength bars with yield strength in excess of 60 ksi shall not be used for design purposes.

8.5	Plastic Moment Capacity for Ductile Concrete Members for SDC B, C & D	The overstrength magnifier of 1.2 for ASTM A 706 reinforcement shall be applied to column plastic hinging moment to determine force demand for capacity protected members connected to a hinging member.
8.6.7	Interlocking Bar Size	Same bar sizes may be used inside and outside of interlocking spirals.
8.8.2	Minimum longitudinal reinforcement	Minimum longitudinal reinforcement of 1% of A_g shall be used for columns in SDC B, C, & D. The minimum Longitudinal reinforcement on top of the shaft shall be the larger of 0.75%A_g of the shaft or 1.0% A_g of the attached column. The minimum longitudinal reinforcement beyond the top of the shaft shall be 0.75%A_g. The clear spacing between longitudinal reinforcement shall not be less than 6" minimum or more than 9" maximum. Longitudinal reinforcement shall be provided for the full length of shaft.
8.8.10	Development length for Column Bars Extended into Oversized Pile Shafts for SDC C & D	Extending column bars into oversized shaft shall be based on either a staggered manner as described in Article 8.8.2, or per current BDM practice based on TRAC Report WA-RD 417.1 “Non Contact Lap Splice in Bridge Column-Shaft Connections” Same size column-shaft is not permissible unless approved by the Bridge Design Engineer.
8.10	Superstructure Capacity design for Integral Bent Caps for Longitudinal direction for SDC B, C & D	The effective width for open soffit girder-deck superstructure as specified in Article 8.10 shall be used instead of current WSDOT practice based on the tributary number of girders per column. The effective width for girder-deck bridges shall be: $B_{eff} = D_c + D_s$, where D_c is the diameter of column and D_s is the depth of superstructure measured from top of column to top of deck including the lower crossbeam. Only girders within the effective width shall be considered for extended strand calculations. In continuous bridges, prestressed girders from adjacent spans shall be the same type and size. Moment induced due to the eccentricity between the plastic hinge location and the center of gravity of the bent cap shall be distributed based on the effective stiffness characteristics of the frame.
8.11	Superstructure Capacity Design for Integral Bent Caps for Transverse Direction for SDC B, C & D	The effective width for overstrength moment calculations M_{po}, shall be taken as $B_{eff} = B_{cop} + 12t$ where t is the thickness of top or bottom slab. Moment induced due to the eccentricity between the plastic hinge location and the center of gravity of the bent cap shall be distributed based on the effective stiffness characteristics of the frame.
8.12	Superstructure Design for Non-Integral Bent Caps for SDC B, C & D	Non-Integral Bent Caps shall not be used for continuous bridges in SDC B, C & D. Bent caps are considered integral if they terminate at the outside of exterior girder and respond monolithically with the girder system under dynamic excitations. Non-integral bent caps such as outriggers, C-bents, etc may be used upon Bridge Design Engineer’s approval.
8.16.2	Cast-in-Place Concrete Piles	Minimum longitudinal reinforcement of 0.75% of A_g shall be provided for CIP piles in SDC B, C, & D. Longitudinal reinforcement shall be provided for the full length of pile.

4.3 Seismic Analysis and Retrofit Design of Existing Bridges

4.3.1 General

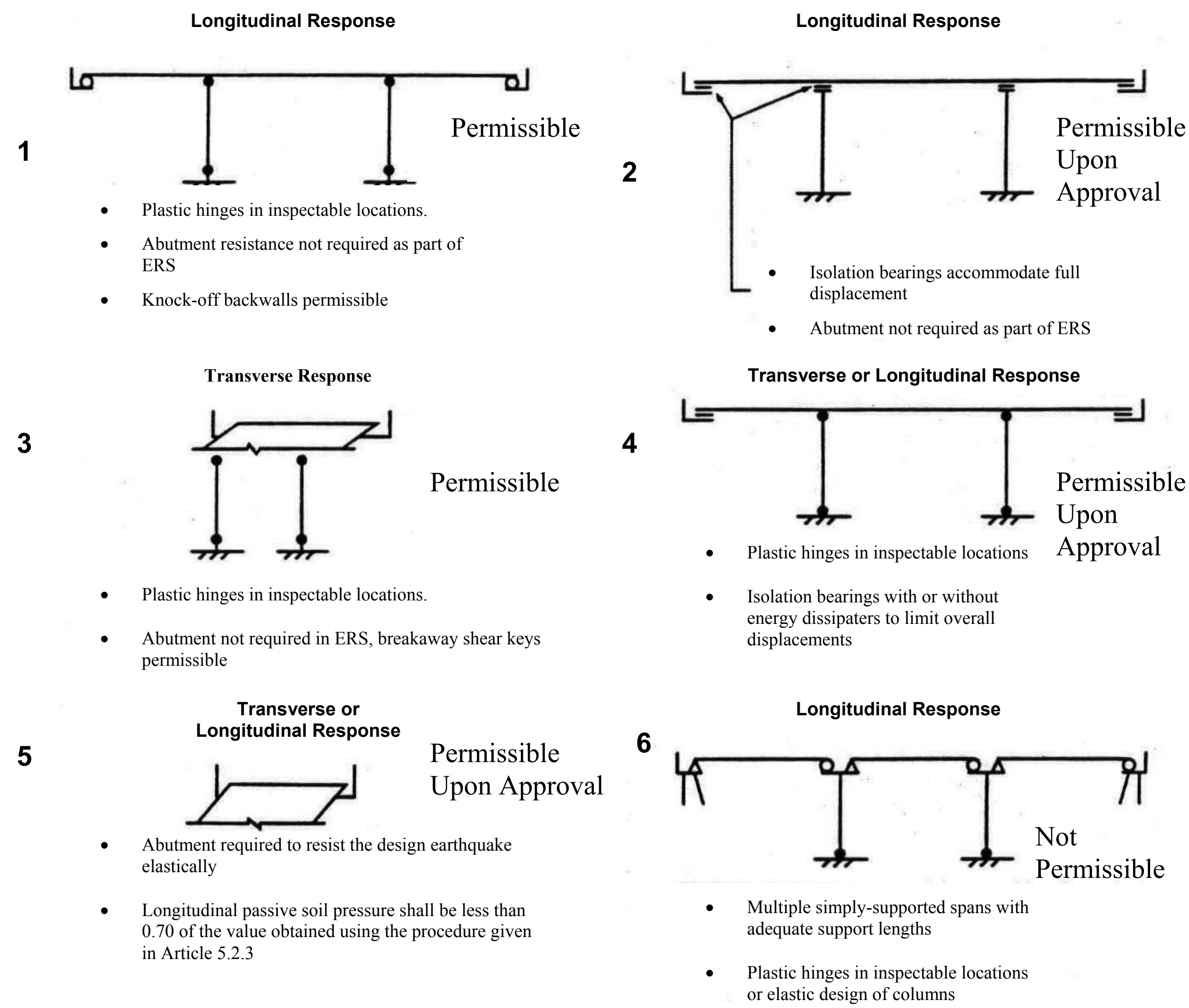
As of January 1, 2008, all seismic analysis and retrofit design for existing bridges shall be performed in accordance with the FHWA publication FHWA-HRT-06-032 “Seismic Retrofitting Manual for Highway Structures: Part 1 – Bridges”.

4.3.2 Seismic Analysis Requirements

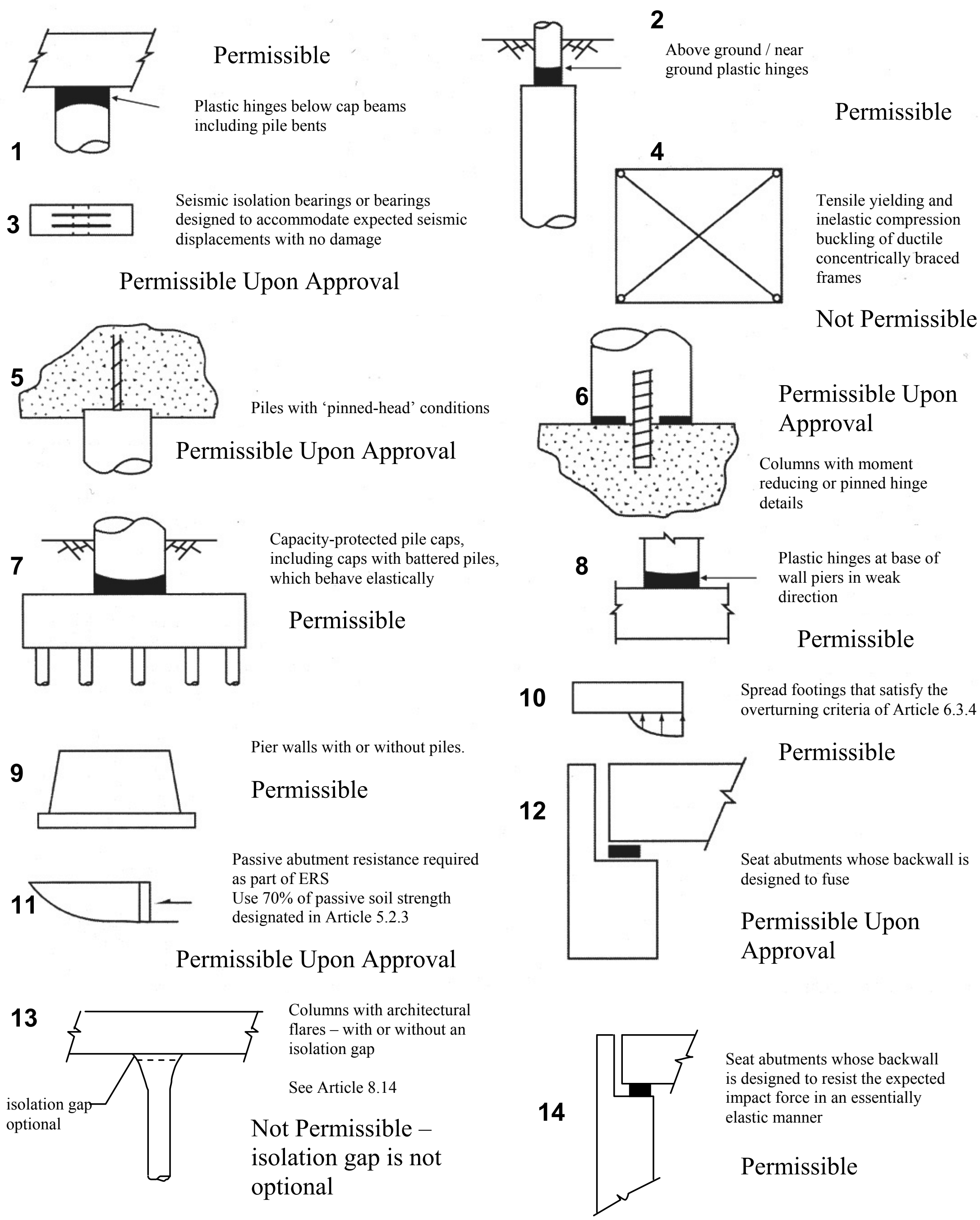
The first step in retrofitting a bridge is to analyze the existing structure to identify seismically deficient elements. The initial analysis consists of generating capacity/demand ratios for all relevant bridge components using Method C – Component Capacity / Demand Method of section 5.4 of the Seismic Retrofitting Manual. In performing this analysis, the seismic demands shall be determined using the Multi-Mode Spectral Analysis of section 5.4.2.2 (at a minimum). The Uniform Load Method of section 5.4.2.1 is not allowed except for use in verifying the results of the Multi-Mode Spectral Analysis. If the results of the Method C analysis indicate that a high level of retrofit is needed, a subsequent analysis, the Method D2 – Structure Capacity/Demand (Pushover) Method of section 5.6 of the Seismic Retrofitting Manual, shall be performed. The displacement demand applied during the pushover analysis shall be the maximum displacement determined from the Method C elastic response analysis. For most WSDOT bridges, the seismic analysis need only be performed for the upper level (1000-year return period) ground motions with a Life Safety seismic performance level.

4.3.3 Seismic Retrofit Design

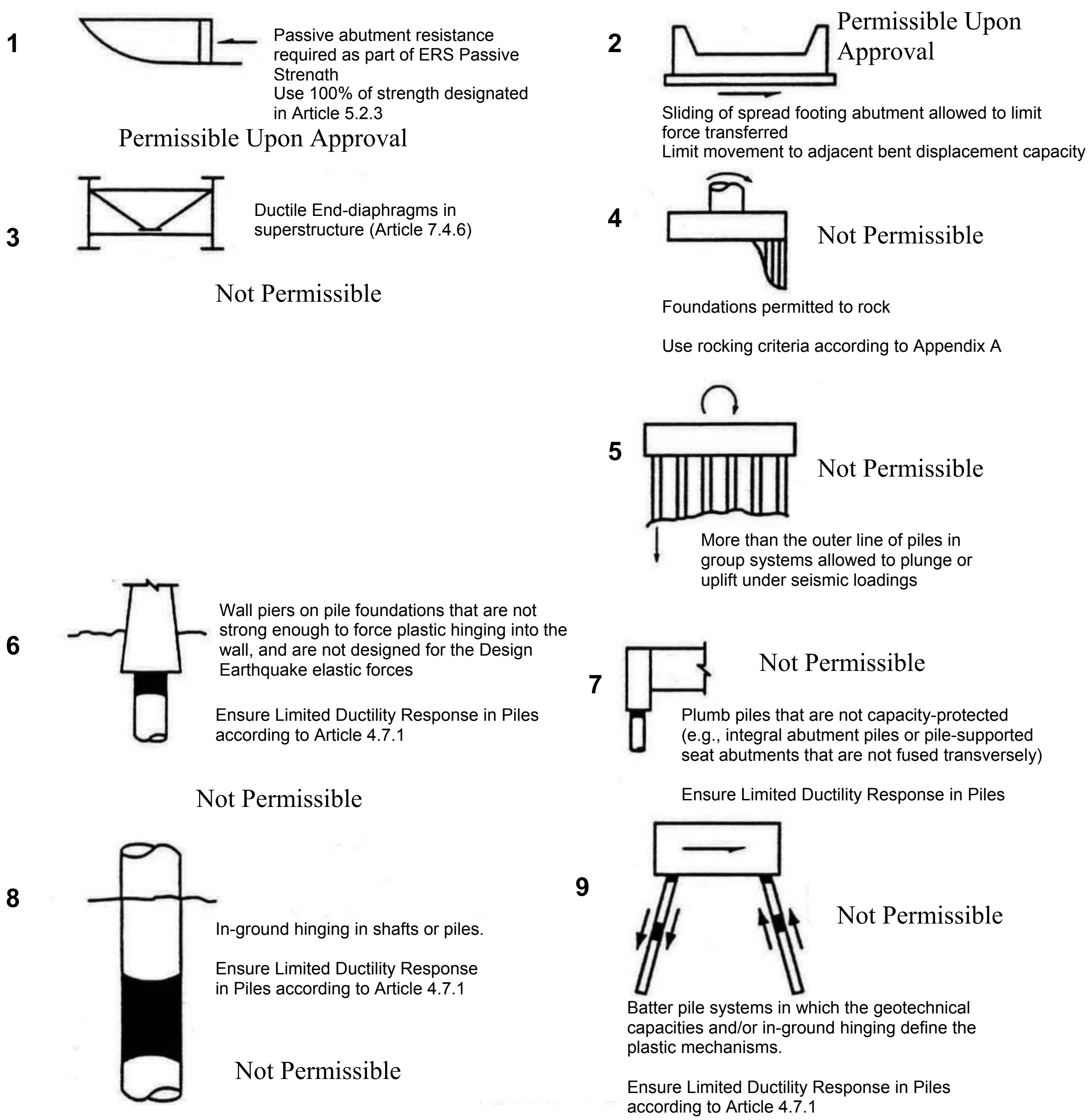
Once seismically deficient bridge elements have been identified, appropriate retrofit measures shall be selected and designed. Table 1-11, Chapters 8, 9, 10, 11, and Appendices D thru F of the Seismic Retrofitting Manual shall be used in selecting and designing the seismic retrofit measures. The WSDOT Bridge and Structure Office Seismic Specialist shall be consulted in the selection and design of the retrofit measures.



Permissible Earthquake Resisting Systems (ERS).
Figure 3.3-1a

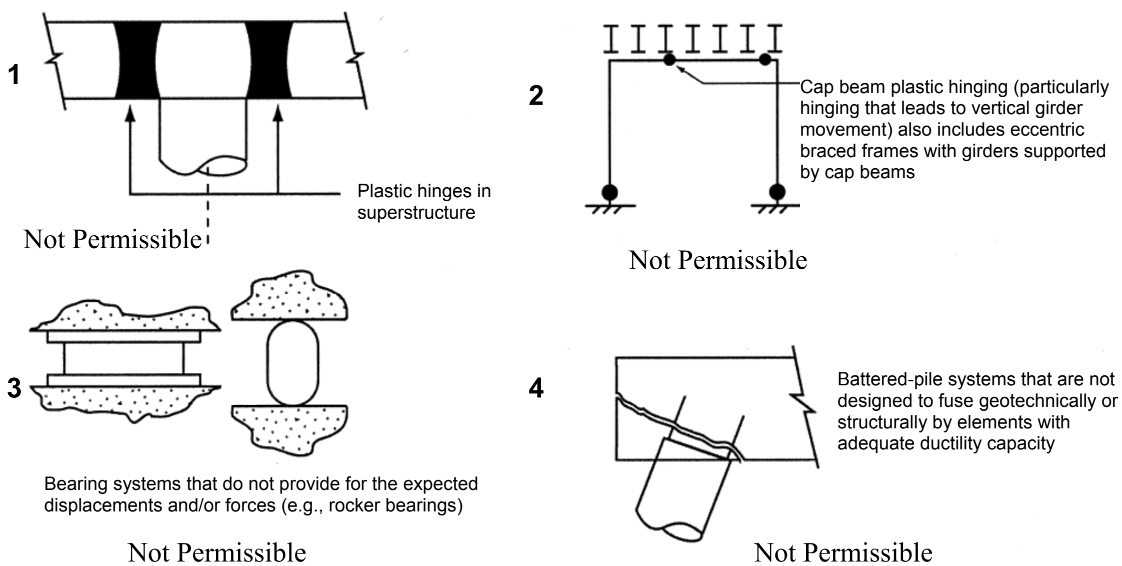


Permissible Earthquake Resisting Elements (ERE)
Figure 3.3-1b



Permissible Earthquake Resisting Elements that Require Owner's Approval (ERE)

Figure 3.3-2



Earthquake Resisting Elements that are not Recommended for New Bridges
Figure 3.3-3

4.3.4 Computer Analysis Verification

The computer results shall be verified to ensure accuracy and correctness. The designer should use the following procedures for model verification:

- Using graphics to check the orientation of all nodes, members, supports, joint and member releases. Make sure that all the structural components and connections correctly model the actual structure.
- Check dead load reactions with hand calculations. The difference should be less than 5 percent.
- Calculate fundamental and subsequent modes by hand and compare results with computer results.
- Check the mode shapes and verify that structure movements are reasonable.
- Increase the number of modes to obtain 90 percent or more mass participation in each direction. SEISAB/GTSTRUDL/SAP2000 directly calculates the percentage of mass participation.
- Check the distribution of lateral forces. Are they consistent with column stiffness? Do small changes in stiffness of certain columns give predictable results?

4.3.5 Earthquake Restrainers

Span unseating is a common problem for existing bridges when the seat width is not sufficient. As a retrofit measure, longitudinal earthquake restrainers are used to tie bridge superstructure sections together at in-span hinges and at locations with expansion joints.

Longitudinal restrainers are high strength bars with both ends anchored on both sides of the adjacent units. A minimum two-inch gap should be maintained at one end of the restrainer to allow for thermal movement. High strength cable may be utilized if the rod cannot fit because of complex geometry, such as: a curved bridge or the movable portion of a ferry terminal.

Bridge Special provision BSP022604.GB6 specifies the current material requirements for the high strength steel bars.

Transverse restrainers are provided to prevent shear failure of the longitudinal restrainers during an earthquake. If the longitudinal restrainers cross a concrete or steel diaphragm, the holes in the diaphragm should be at least one inch larger than the diameter of the high strength steel bars. The transverse restrainer shall limit the bridge transverse movement to less than ½ inch.

A satisfactory method for designing the size and number of restrainers required at expansion joints is not currently available. Adequate seat shall be provided to prevent unseating as a primary requirement. For retrofit, earthquake restrainers shall be designed in accordance with the Caltrans Equivalent Static Analysis method and checked with AASHTO LRFD Section 3.10.9.5.

4.99 Bibliography

AASHTO LRFD Bridge Design Specifications, 4th Edition, 2007 and Successive Interims.

AASHTO Guide specifications for LRFD Seismic Bridge Design, May, 2007.

Caltrans, 1989, Bridge Design Aids-Equivalent Static Analysis of Restrainers, California Department of Transportation, Sacramento, California, pp 14-11 to 14-25.

FHWA *Seismic Retrofitting Manual for Highway Structures: Part 1-Bridges*, Publication No. FHWA-HRT-06-032, January 2006.

Appendix 4.1-A1 — Earthquake Probability - Poisson Model

Earthquake Probability Example

The probability, P, that an earthquake can occur within a certain time frame, t_L, can be estimated using Poisson's distribution:

$$P = 1 - e^{-\lambda_a t_L}$$

For example, assume the average return time or recurrence of an earthquake is 100 years, estimate the probability that it will occur in the next 100 years.

Let T_a = mean return period in years = $1/\lambda_a$

Where: λ_a = average annual probability that the peak ground acceleration will exceed a certain acceleration, "a".

In a typical design situation, the designer is interested in the probability that such a peak exceeds "a" during the life of the structure, t_L.

For the earthquake recurrence example, T_a = 100 years, λ_a = 1/100 = 0.01 and t_L = 100 years:

$$P = 1 - e^{-\lambda_a t_L} = 1 - e^{-0.01(100)} = 0.63 \text{ or } 63\%$$

Using the same earthquake, determine the chance that the same earthquake will occur within the next 20 years:

$$P = 1 - e^{\lambda_a t_L} = 1 - e^{-0.01(20)} = 0.18 \text{ or } 18\%$$

An earthquake with a peak ground acceleration coefficient map with a 7% probability of exceedance in 75 years corresponds to a return period of 1000 years.

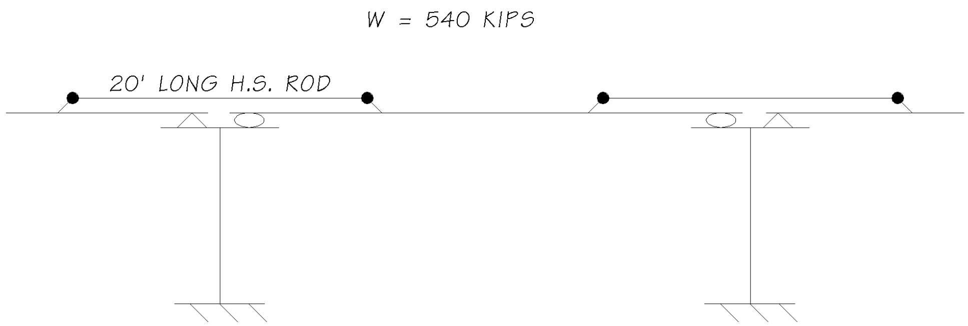
Proof: T_a = 1000 years, λ_a = 1/1000 and t_L = 75 years

$$P = 1 - e^{\lambda_a t_L} = 1 - e^{-(1/1000)(75)} = 0.0723 \text{ or } 7\% \text{ Checks}$$

Earthquake Restrainer Example

Bridge Type: Multiple Simple Spans

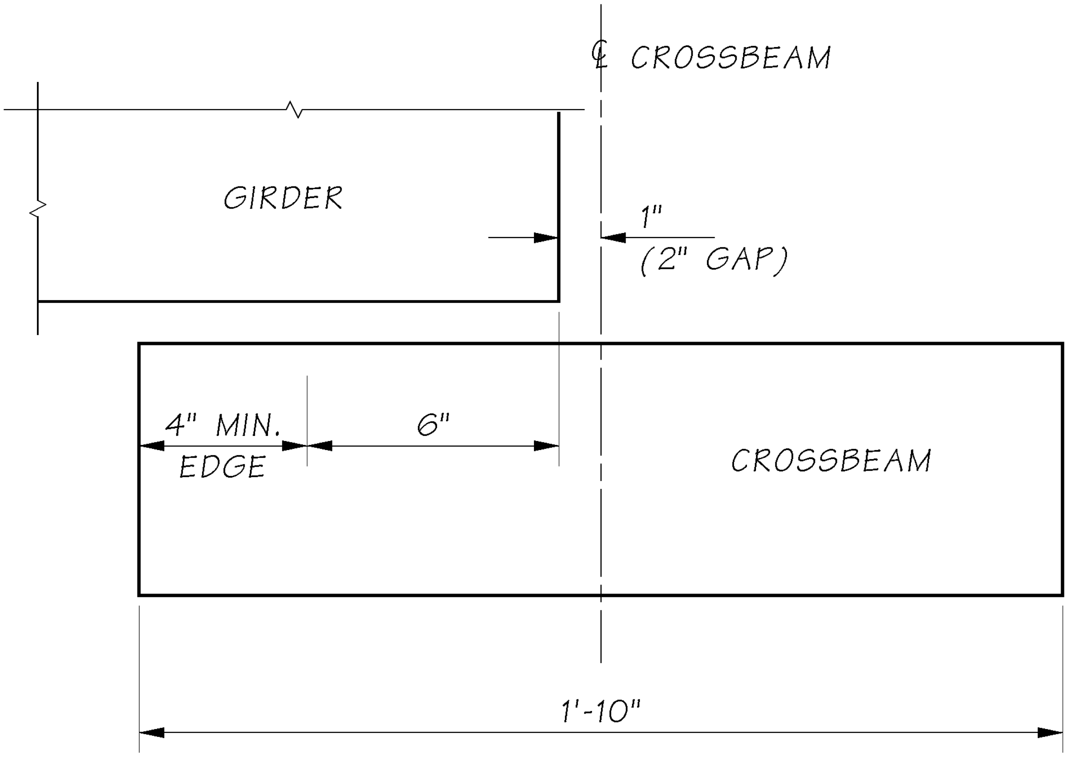
This Design Example is based on CALTRAN's *Seismic Design References* (1997)



Seismic Data: Acceleration Coeficient, A = 0.3g; Soil Type II, S = 1.2
Dead Load of the Span = 540 kips

Bearings: Roller Bearings with no longitudinal restraint. Shear blocks to be added to provide transverse restraint.

Restrainers: 20 foot long High-Strength steel rods (ASTM F1554 Grade 105)
F_y = 105 ksi and E = 29,000 ksi
2 inch gap at end of High-Strength rod



<u>Calculate Available Seat Width</u>: (22”/2) – 4” –1” = 6 inches

<u>Determine Maximum Restrainer Deflection (D_r)</u>:

Let D_y = max. elastic deformation of rod when restrainer is stressed to F_y

$D_y = F_yL/E$ = (105 ksi)(20ft)(12in/ft)/(29,000 ksi) =	0.9 inches
D_{gap}	<u>2.0</u>
D_r = Resultant Longitudinal Displacement = $D_y + D_{gap}$ =	2.9 inches < 6 inches

<u>Try four 1 inch diameter rods</u>: $A_g = 4(0.785 in^2) = 3.14$ in^2 Use A_g of plain rod for stiffness/ elongation calculations and use tensile area, A_t, for stress check.

(Note: $A_g = A_t$ if a high strength rod is threaded for its full length):

Calculate the stiffness, K_t ,provided by the restrainer rods:

$$K_t = \frac{F_y\left(A_g\right)}{D_r} = \frac{105(3.14)}{2.9} = 114 kips/inch$$

Calculate the period, T:

$$T = 2\rho\sqrt{\frac{W}{gk_t}} = 0.32\sqrt{\frac{540}{114}} = 0.70 \sec onds$$

where: T = period in seconds
W = Dead Load of the span = 540 kips
g = 32.2 ft/sec^2 x 12in/ft = 386 inches/sec^2
K_t = 114 kips/inch

Calculate the Elastic Seismic Response Coefficient, C_s, for Multimodal Analysis:

$$Cs = \frac{1.2AS}{T^{2/3}} = \frac{1.2(0.30)(1.2)}{0.70^{0.67}} = 0.5g$$

when A ≥ 0.30g, Cs need not exceed 2.0A

Therefore, C_s = 0.55g < 0.6g, okay

Calculate the seismic force and tensile stress, f_t, to be resisted by the restrainers:

Use tensile area: $A_t = 0.606\ in^2$ per restrainer rod

Seismic Force = C_sW = 0.55(540) = 297 kips

$$f_t = \frac{C_sW}{A_t} = \frac{297}{4(0.606)} = 122.5ksi > 105ksi$$

No Good for Stress

Calculate the elastic elongation in the four 1 inch diameter retrainer rods, D_t:

$$D_t = \frac{C_sW}{K_t} = \frac{297}{114} = 2.6inches < D_r = 2.9inches$$

okay

The elastic elongation of the restrainers is less than the resultant displacement. However, the tensile stress at the threaded ends of the rod exceeds fy. Therefore, it is necessary to increase the number of restrainers or increase the diameter of the restrainers in order to reduce the elastic elongation.

Try four 1-1/8 inch diameter x 8UN threaded rods: $A_g = 4(0.994\ in^2) = 3.98\ in^2$

$$K_t = \frac{F_y(A_g)}{D_r} = \frac{105(3.98)}{2.9} = 144kips/inch$$

$$T = 2\rho\sqrt{\frac{W}{gk_t}} = 0.32\sqrt{\frac{540}{144}} = 0.62\sec onds$$

$$Cs = \frac{1.2AS}{T^{2/3}} = \frac{1.2(0.30)(1.2)}{0.62^{0.67}} = 0.60g = 0.6g$$

$$D_t = \frac{C_sW}{K_t} = \frac{0.6(540)}{144} = 2.25inches < D_r = 2.9inches$$

okay for Elongation

$$f_t = \frac{C_sW}{A_t} = \frac{324}{4(0.790)} = 102.5ksi < 105ksi$$

okay for Stress

Use four 1-1/8 inch diameter x 20 ft long ASTM F1554 Grade 105 High-Strength Rods with Fy = 105 ksi. Specify a Charpy V-Notch (CVN) of 25 ft-lbs @ 40°F, or Supplemental Requirement S5 (15 ft-lbs @ -40°F.***BRIDGE DESIGN MANUAL***

Circular Column Steel Jacket Retrofit Example

Lateral tie reinforcement of #4 bars at 12" centers is inadequate confinement for the longitudinal column reinforcement.

Concrete core is adequate to resist seismic transverse shear force

The column is 3 ft. in diameter. Assume clearance is 1" between column and steel jacket.

Determine thickness of steel jacket.

Using the FHWA Guidelines from *Seismic Retrofitting Manual for Highway Bridges*, (1995):

$$t = \frac{f_{cc} D}{58}$$

where: t = thickness of steel jacket in inches
f_{cc} = confining concrete core pressure in ksi = 0.300 ksi
D = 36" + 2" = 38"

$$\therefore t = \frac{(0.3)(38)}{58} = 0.20" > 0.25" \text{min}$$

Use 1/4" thick steel jacket with Fy = 36 ksi

Lateral tie reinforcement of #4 bars at 12" centers is inadequate confinement for the longitudinal column reinforcement.

Concrete core is adequate to resist seismic transverse shear force

The column is 5 ft. in diameter. Assume clearance is 1" between column and steel jacket.

Determine thickness of steel jacket.

Using the FHWA Guidelines from *Seismic Retrofitting Manual for Highway Bridges*, (1995):

$$t = \frac{f_{cc} D}{58}$$

where: t = thickness of steel jacket in inches
f_{cc} = confining concrete core pressure in ksi = 0.300 ksi
D = 60" + 2" = 62"

$$\therefore t = \frac{(0.3)(62)}{58} = 0.32" > 0.25" \text{min}$$

Use 3/8" thick steel jacket with Fy = 36 ksi

A seismic analysis shows the 4 ft. diameter column is required to undergo a plastic drift angle of 0.045 radians.

The existing lateral confining reinforcement is inadequate.

Longitudinal bars are #11 Grade 40 reinforcement and $\rho_1 = 0.04$ or 4 %.

The ratio $\frac{P}{f'_{ca} A_g} = 0.2$

where: P = resultant axial force in kips
$f'_{ca} = 1.5(f'_c) \approx 5$ ksi for an original concrete design strength of 3,000psi
Ag = gross concrete column area in in^2

Determine $\frac{t_j}{D} \geq 0.01$ from Figure 8.5(a) *Seismic Design of Bridges*, Priestley, Seibel, and Calvi (1996), p. 592

$\therefore t_j = 0.01(48+2) = 0.50''$

<u>Use ½" thick steel jacket with Fy = 36 ksi</u>

Lateral tie reinforcement of #4 bars at 12" centers is inadequate confinement for the longitudinal column reinforcement.

Concrete core is adequate to resist seismic transverse shear force

The size of the rectangular column is 2' x 6'

Check size of ellipse to provide 1" clearance between column and steel jacket.

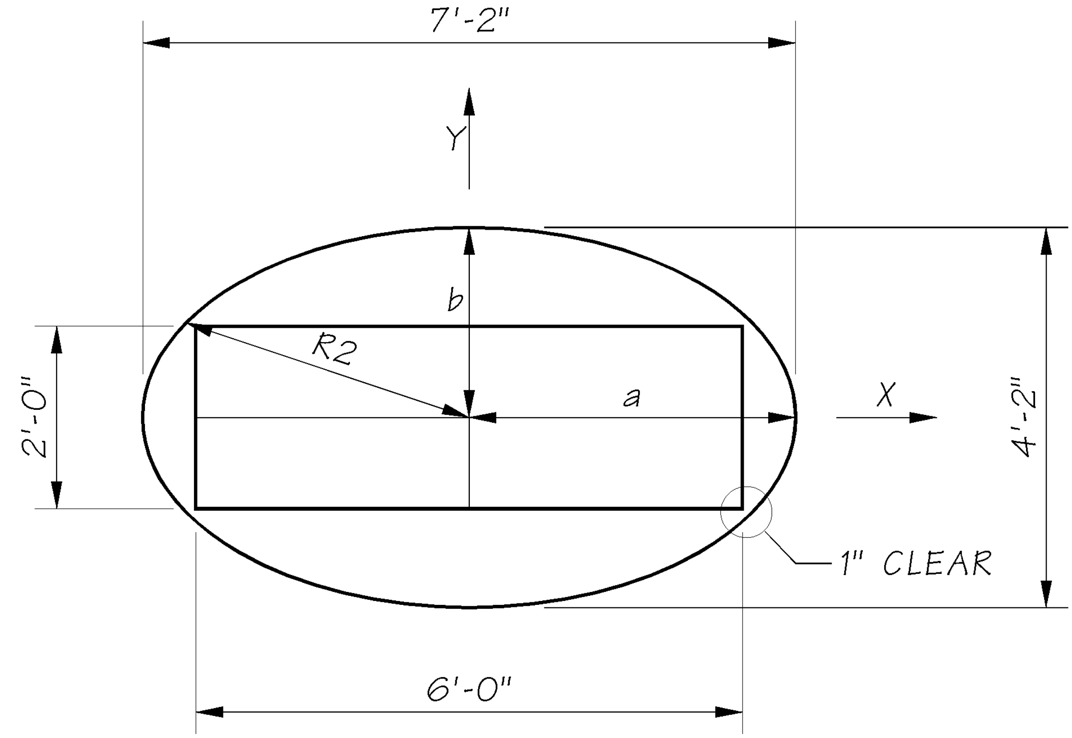
$$\frac{x^2}{a^2} + \frac{y^2}{b^2} = 1$$

After several tries, use an elliptical shape such that:

Long axis = 7'-2" such that $a = 3'-7''$ and

Short axis = 4'-2" $b = 2'-1''$

Find equivalent diameter, D = 2a = 86”



Using the FHWA Guidelines from *Seismic Retrofitting Manual for Highway Bridges*, (1995):

$$t > \frac{f_{cc}D}{58}$$

$$\therefore t = \frac{(0.3)(86")}{58} = 0.44"$$

Use ½” thick steel jacket with Fy = 36 ksi

Lateral tie reinforcement of #4 bars at 12” centers is inadequate confinement for the longitudinal column reinforcement.

Concrete core is adequate to resist seismic transverse shear force

The size of the rectangular column is 4’ x 6’

Check size of ellipse to provide 1” clearance between column and steel jacket.

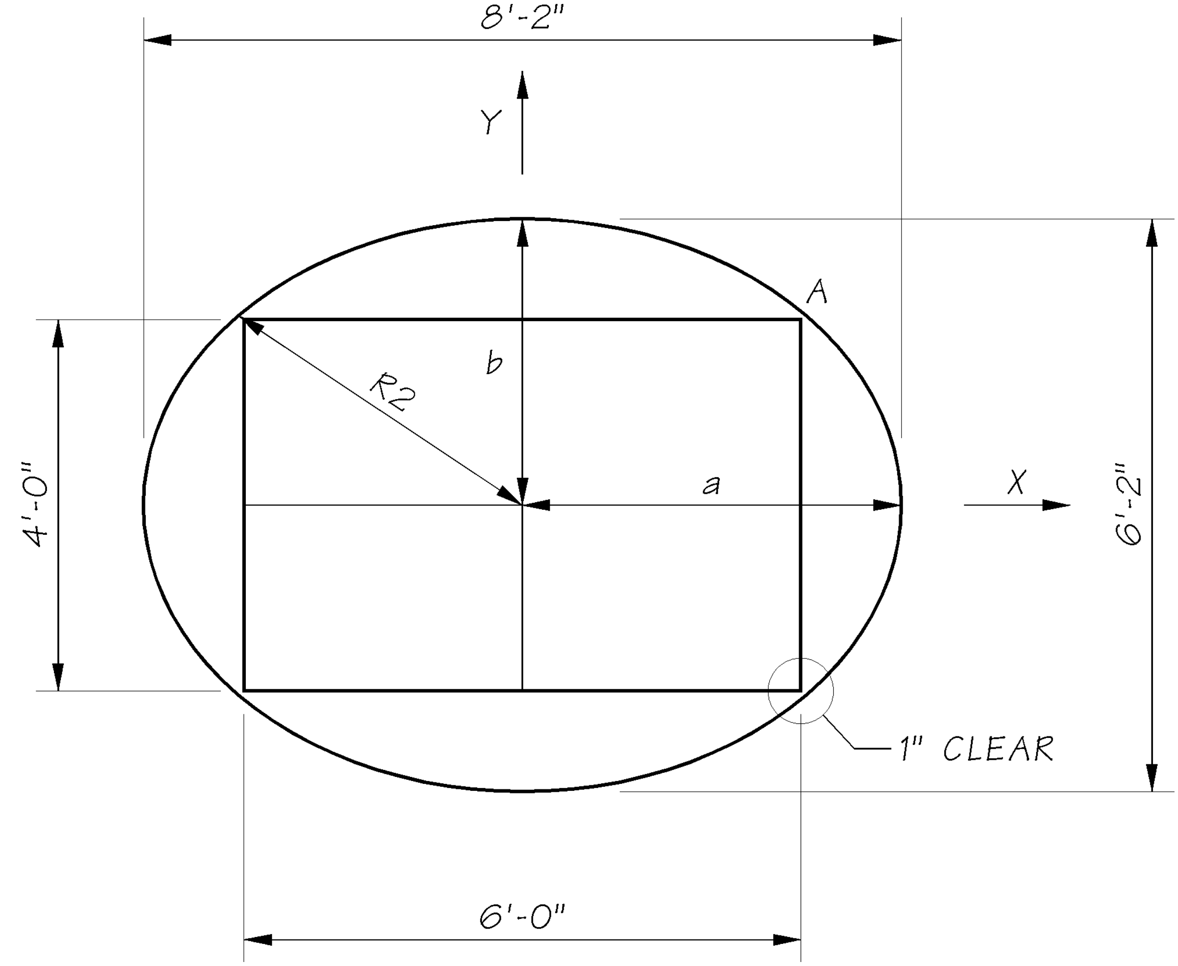
$$\frac{x^2}{a^2} + \frac{y^2}{b^2} = 1$$

After several tries, use an elliptical shape such that:

Long axis = 8’-2” such that $a = 4'-1"$ and

Short axis = 6’-2” such that $b = 3'-1"$

Find equivalent diameter, D = 2a = 98”



Using the FHWA Guidelines from *Seismic Retrofitting Manual for Highway Bridges*, (1995):

$$\therefore t = \frac{(0.3)(98")}{58} = 0.51"$$

Use ½” thick steel jacket with Fy = 36 ksi